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BOND STRENGTH OF CFRP AND STEEL BARS IN CONCRETE AT ELEVATED TEMPERATURE

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Abstract: Novel concrete elements are emerging utilizing high performance self-consolidating concrete (HPSCC) reinforced with high-strength, lightweight, and non-corroding carbon fiber reinforced polymer (CFRP) prestressed reinforcement. The fire performance of these elements must be understood before they can be used with confidence. In particular, the bond performance of the novel CFRP reinforcement at elevated temperatures requires investigation. This paper examines the bond performance of a specific type of CFRP tendon as compared with steel prestressing wire. The results of transient elevated temperature bond pullout and tensile strength tests on CFRP tendons and steel prestressing wire are presented and discussed, and show that bond failure at elevated temperature is a complex phenomenon which is influenced by a number of interrelated factors, including the type of prestressing, degradation of the concrete, CFRP, and steel, differential thermal expansion, thermal gradients and stresses, release of moisture from the concrete, and loading. It is shown that CFRP tendons are more sensitive to bond strength reductions than to reductions in tensile strength at elevated temperature.

Keywords: Bond strength, composite materials, DMA, elevated temperatures, carbon FRP, prestressing reinforcement, pullout tests, steel wire, TGA.

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INTRODUCTION

The use of fiber reinforced polymers (FRPs) in civil engineering applications has been developing for more than 40 years. FRPs are now widely used as externally bonded FRP sheets and plates to increase the shear, flexural and/or axial strength of deficient reinforced concrete elements. Another promising and reasonably widely implemented application of FRPs is their use as partial or total replacement of steel reinforcement in new concrete structures.

The use of internal FRP reinforcement for prestressing of concrete structures is motivated predominantly by a desire to prevent electrochemical corrosion of the reinforcement and thus to build more durable structures. Additionally, however, a desire to build more sustainable and durable structures has made careful selection, design, and optimization of both the concrete mixtures and reinforcing materials commonplace. These considerations have promoted the emergence of elements which incorporate high performance self-consolidating concrete (HPSCC) with novel prestressing materials, such as carbon fiber reinforced polymer (CFRP) tendons which are high-strength, creep resistant, lightweight, non-corroding, and magnetically invisible. One example of this is a novel type of HPSCC precast panel with pretensioned carbon fiber reinforced polymer (CFRP) tendons; this is now being implemented in load-bearing panels for building envelopes [1]. Several field projects have been realized in recent years making use of thin-walled structural elements made of high performance concrete prestressed with CFRP tendons [1]. One notable example of this is the use of precast, prestressed 8.8 m (28.87 ft) long beams with L-shaped cross-sections, only 60 mm (2.36 in.) thick in an innovative project in Switzerland [2] (Figure 1).

Designing precast pretensioned concrete elements by combining the advantages of HPSCC and CFRP bars makes it possible to manufacture thin-walled, lightweight, fatigue resistant, and durable HPSCC precast elements, despite the relatively high initial costs of FRP prestressing bars in comparison to prestressing steel.

The performance of these HPSCC precast concrete elements in fire is not, however, well known, and a better understanding of their response to fire must be developed before they can be used with confidence in the wide variety of structural applications for which they might be considered. Numerous studies and past experience have shown that conventional steel reinforced or prestressed concrete structural elements generally exhibit good performance in fires. This is due primarily to the inherent and beneficial self-insulating characteristics of concrete [3], which protects the internal steel reinforcement from the heat of the fire. For reinforced or prestressed concrete elements, adequate structural fire resistance is typically ensured by prescribing a minimum allowable concrete cover depth to the steel reinforcement/prestressing. In the case of CFRP prestressed HPSCC elements such as those described above, two additional issues become critical and are the focus of the current paper:

1. Because there are essentially no concerns regarding corrosion of the CFRP reinforcement, the concrete cover can be considerably less in CFRP reinforced or prestressed elements than in steel prestressed elements; this allows thin-walled elements without increased risk of corrosion.
2. Reductions of the bond strength between steel reinforcement and concrete are not generally considered to be a governing factor dictating the fire resistance of steel reinforced or prestressed concrete elements. It is widely thought, however, that that degradation of the bond between FRP bars/tendons and concrete at elevated temperature is considerably more critical than loss of the reinforcement's tensile strength [4]. FRP reinforcements' bond strength reductions are widely thought to be the limiting factor for the fire-safety of FRP reinforced/prestressed concrete [4], although the magnitudes of bond strength reductions and their impacts on the load-bearing capacity of heated FRP reinforced/prestressed concrete structures remain largely unknown.

RESEARCH SIGNIFICANCE

The bond between both FRP and steel reinforcing bars (prestressed and non-prestressed) and concrete deteriorates at elevated temperature [5], [6]. The ultimate tensile strength of both steel and FRP bars/tendons is also reduced by exposure to elevated temperatures; these reductions are now well known for steel reinforcements, as defined by Eurocode 2 [7] for example, but remain largely unknown for FRP reinforcements [8].

National design codes [9], [10], [11], [12] focusing on the design of FRP reinforced concrete structures all assume perfect bond between the FRP reinforcement and the concrete for ambient temperature analysis and design. The bond strength capacity of FRP reinforcing or prestressing bars relies primarily on the strength and stiffness of the polymer resin matrix at the bars' surface (which typically incorporates a sand coating, spiral fibre roving, and/or a ribbed shaped resin). The polymer resin at the surface of the bars softens at temperatures of less than 150 ºC (302 °F) for most available FRP reinforcing bars. The glass transition temperature (T_g) of the polymer resin is widely used to define the limiting temperature at which degradation of the tensile strength of composites materials occurs [10], [13], yet the research to support such an approach is relatively scarce. Furthermore, despite the known importance of bond strength reductions for FRP bars in concrete, relatively little attention has been given to defining how the degradation of the polymer resin matrix affects the bond strength capacity at elevated temperatures or in fire; this is the focus of the current paper.

EXPERIMENTAL INVESTIGATION

An experimental program was undertaken, consisting of transient elevated temperature bond pullout and tensile strength tests on CFRP prestressing tendons and deformed steel prestressing wires of comparable strength and stiffness. The following sections provide details of the specific materials and test methods used.

Materials

CFRP prestressing tendons — Round, pultruded, sand-coated CFRP prestressing tendons supplied by SACAC Ltd[1], Switzerland, were used in this study. The carbon fiber volume fraction was 62% and the bars had an epoxy polymer resin matrix. The CFRP bars had a 5.4 mm (0.213 in.) nominal diameter and were fabricated by the pultrusion process [9]. To promote a better bond with concrete, a sand coating was applied to the surface of the bars by broad-casting silica sand into a coating of epoxy resin applied after the primary pultrusion and curing process. The sand coating was approximately 0.3 mm (0.012 in.) thick. The CFRP bar's manufacturer specified nominal tensile strength was 2000 MPa (290 ksi), with an elastic modulus of 150 GPa (21,756 ksi). The tensile

[1] Please note that the specific CFRP and steel prestressing tendon suppliers are noted solely for the purposes of factual accuracy.

stress-strain relationship for the CFRP is linear-elastic to failure, with an ultimate strain of 1.33%. The bars have a nominal mass of 0.123 kg⁄m (0.0827 lb/ft), which is about half the value of the comparative steel prestressing wire used in the current project. A sample of the CFRP prestressing tendon is shown in Figure 2(a).

Steel prestressing wire — For the purposes of comparison, deformed steel prestressing wire produced specifically for prestressing applications by NEDRI Spanstaal BV was also used in this study. The steel reinforcement was 6 mm (0.236 in.) in nominal diameter with a nominal mass of 0.221 kg⁄m (0.149 lb/ft). Ribs were applied on the surface of the wire during the cold working manufacturing process. The manufacturer's specified design yield strength (0.2% offset) was 1592 MPa (231ksi) and the ultimate strength was 1770 MPa (257 ksi), with an elastic modulus of 210 GPa (30,458 ksi). The yield strain was 0.76% and the ultimate strain was 5.4%. A sample of the steel wire is shown in Figure 2(b).

Concrete (HPSCC) — The high quality and comparatively high cost of CFRP reinforcements requires a correspondingly high quality concrete mixture to justify use of CFRP prestressing tendons. A high performance, self-consolidating, polypropylene fiber reinforced concrete (HPSCC) with a 28-day minimum cylinder strength of 75 MPa (10.9 ksi) was used in this study. The details of the concrete mixture are proprietary, but it included normal Portland cement, silica fume, fly ash, polypropylene microfibers, super-plasticizer, and a precise grain size distribution of selected 0-8 mm limestone aggregates.

Reinforcements' bond mechanisms

In reinforced concrete elements, the longitudinal tensile forces transferred from the reinforcement to the surrounding concrete are influenced by the reinforcement's surface characteristics and shape, the concrete's tensile and compressive strength, concrete cover, clear spacing between reinforcing bars, casting orientation, load-time history of the structure, and the reinforcement's elastic modulus, Poisson's ratio, and diameter [14].

When deformed steel reinforcement is used, the ribbed surface of the bar interacts with concrete by chemical adhesion, frictional forces, and mechanical anchorage (given by bearing of the ribs against the concrete surrounding the reinforcement). Elevated temperature bond strength reductions for deformed steel reinforcements are most likely to be caused by loss of chemical adhesion and reductions in concrete strength at elevated temperature.

When CFRP reinforcement with a sand-coated epoxy resin surface is used, bond to concrete is also given by chemical adhesion, frictional forces, and a diminished mechanical anchorage, relative to the one in deformed steel, given by the sand coating's silica particles which are chemically and mechanically anchored in the reinforcement's surface resin layer. It is the loss of strength and stiffness of the resin at the surface of the reinforcement at elevated temperature which appears to be critical. For commercially available FRP reinforcing bars, several supplementary methods have been developed to improve bond with concrete under ambient conditions – for example deformation at the bar's surface by wrapping additional resin-saturated fibers in a spiral around the bar. This paper considers only sand-coated CFRP prestressing tendons without additional surface deformations.

It should be noted that, both at ambient temperature and under fire conditions, the bond between steel reinforcements (prestressed and non-prestressed) and concrete is not fully understood and remains a topic of research interest. Nonetheless, the bond between steel and concrete is typically considered to be perfect (even under fire conditions), even if this is clearly not the case in real structures during fire [15].

Pullout test methods and procedures

To study the load transfer and bond between concrete and internal reinforcement, previous researchers [5], [16] have used strain gauges placed at the concrete-reinforcement interface in large or mid-scale reinforced concrete elements, along with other techniques employed to measure stress and relative displacement on both the reinforcement and the concrete. These approaches, while useful for understanding load transfer in real structures, are time-consuming and costly. Bond pullout tests present an economical and simple testing option for comparative evaluation of bond performance of different types of reinforcing bars within concrete, despite the fact that the stress conditions encountered in reinforced concrete elements differ considerably from those encountered in pullout tests [17], [18]. For instance, pullout tests have been used by previous researchers to compare the bond performance of steel and FRP reinforcing bars during (or after) exposure to elevated temperatures [6], [19].

Abbasi and Hogg [19] studied the bond strength of glass FRP reinforcing bars in concrete at temperatures in the range of 25-120 °C (77-248 °F) after long-term exposure to an alkaline environment. Their tests consisted of steady state heat-then-load bond pullout tests on glass FRP bars embedded over a length of 60 mm (2.36 in.) within 100 mm (3.96 in.) cubes of concrete. The specimens were heated in an oven until they reached a pre-determined temperature between 20 °C (68 °F) and 120 °C (248 °F) and were then loaded to failure. Abbasi and Hogg [19] concluded that degradation of bond strength with temperature obeys a similar relationship irrespective of prior conditioning of the specimens, and that the nature of the polymer resin matrix at the surface of the FRP bars determines the magnitude and rate of degradation bond strength. In their study, average bond pullout strengths at 120 °C (248 °F) were about 57% of ambient temperature values. These data are not particularly helpful, however, since specific information on the deterioration in mechanical properties of the resin with elevated temperature is not given. It should also be noted that steady state (heat-then-load) tests are less representative of the in-service conditions for FRP bars in concrete than transient (load-then-heat) tests, particularly for prestressed FRP reinforcement which is under considerable sustained stress while a structure heats during a fire.

Katz et al. [6] tested five different types of FRP bars using transient (load-then-heat) elevated temperature bond pullout tests under sustained load. These were executed by loading specimens up to a predefined bond stress level and then heating the specimen under constant applied load until failure occurred. The test specimens consisted of FRP bars embedded in 150 mm (5.91 in.) diameter concrete cylinders over a length of 60 mm (2.36 in.). Heating was applied by a heating element wrapped around the cylinders' circumference. Katz et al. [6] observed a severe reduction in the bond strength (in some cases as high as 75%) as the temperature was raised above the glass transition temperature of the bars' polymer resin matrices (which ranged between 60 °C (140 °F) and 124 °C (255 °F)). They also noted that a thick polymeric layer at the surface of the FRP bar negatively affected the bond both at ambient temperature and particularly at elevated temperature. It should be noted that during their initial tests, Katz et al. [6] encountered "premature" failures of the bond, which were attributed to "flux of vapor outward at high pressure which stimulated the slippage of the bar." They artificially prevented these failures in subsequent tests by pre-drying the specimens in ovens at 100 °C (212 °F). This approach is non-representative of the thermal conditions that would be encountered in a real fire, and Katz et al.'s results should be viewed with this in mind given that it appears that moisture plays an important role in influencing bond behaviour at elevated temperature.

The research performed for the current paper was performed in a similar manner as Katz et al.'s testing [6], with the notable exception that the specimens were not artificially pre-dried prior to testing. Testing was performed by loading the specimens (as described below) up to a predefined bond stress and then heating them until failure occurred under sustained load. Heating was accomplished using a silicone rubber heating pad wrapped around the specimens' circumference (Figure 3). A total of 16 specimens were tested, eight with embedded CFRP tendons and eight with embedded steel prestressing wires.

The pullout specimens in the current study, which are shown schematically in Figure 3, consisted of concrete cylinders with diameters of 100 mm (3.96 in.) and lengths of 250 mm (9.84 in.). Each cylinder had a single embedded prestressing bar (CFRP or steel). The casting moulds were designed in such way that the reinforcement was placed vertically and axisymmetrically. Bond breaker was placed over the top (50 mm (1.97 in.)) and bottom (40 mm (1.57 in.)) of the reinforcing bars to allow for a bonded length of 160 mm (6.30 in.). The bond breaker also prevented the development of localized artificial confinement of the bonded length due to compressive load on the concrete at the loaded end, and promoted axisymmetric heat transfer along the bonded length to assure as uniform a bond line temperature as possible. The bonded length was chosen as 160 mm (6.30 in.) because this was determined to be the prestress transfer length for the CFRP tendons at ambient temperature as measured through prior research [20]. Special consideration was given to the procedure used for accurately measuring of the slip of the reinforcement at both the loaded and free ends of the cylinders – a novel digital image correlation analysis technique was used (Figure 3) [21], [22]. The image correlation technique was verified using conventional linear potentiometers (LPs), which were also used to measure slip at both ends. The precision of this measurement technique has been proven to be better than 1/10th of one pixel [23], and the linear pixel density achieved within the digital images collected during the tests was 35 pixels per mm (889 pixels per inch).

Temperatures were measured at various locations within the test specimens using T-type thermocouples. Three thermocouples were fixed to each of the CFRP tendons and steel wires at the bottom, middle and top of the 160 mm (6.30 in.) bonded lengths. The thermocouples were placed using a small amount of common instant adhesive, and the effect of the thermocouple installation on the bond strength is considered to be negligible. Four additional thermocouples were placed at the cylinder's perimeter on opposite sides at its third points (Figure 3). When displaying pullout test's results, the concrete-bar interface temperature is shown as the mean of the three thermocouples placed at the concrete-bar interface. The temperature at the surface of the outer concrete cylinder is taken as the average from the four thermocouples placed at the interface between the concrete and the heating wrap (Figure 4).

During initial scoping testing it was determined that the bond behavior of the specimens during heating was far more complex, and depended on many more factors, than had initially been expected. The result of this realization was that three distinct pullout test procedures/conditions were used. These depended on the thermal exposure, the initial load condition, and the failure condition. The following three testing conditions were used:

- *AT (Ambient Temperature)* — Specimens were loaded at ambient temperature, under displacement control at a rate of 1.19 mm/min (0.05 in./min), until pullout failure occurred.
- *SL (Sustained Loading)* — Specimens were loaded at ambient temperature, under displacement control at a rate of 1.19 mm/min (0.05 in./min), up to a predefined sustained load. The load was then maintained, under load control, and the temperature of the heating wrap was increased at 24 °C/min (43 °F/min) until failure occurred.
- *IL (Increased Loading)* — Specimens were loaded at ambient temperature, under displacement control at a rate of 1.19 mm/min (0.05 in./min), up to a predefined sustained load. The load was then maintained, under load control, and the temperature of the heating wrap was increased at 24 °C/min (43 °F/min) (Figure 4) in an attempt to induce failure. When no failure occurred as temperature of the specimen increased and a steady state temperature was reached that could not be exceeded given the heating technique used, the load was increased, again under displacement control, until pullout failure occurred. In these tests the reinforcement temperature exceeded 160 °C (320 °F) (at about 230 minutes) at the time that load was increased causing failure.

The specimen notation used in the tables and figures which follow is defined by four parameters: (1) tendon type (C for CFRP or S for steel), (2) pullout test condition (AT, SL, or IL as discussed above), (3) sustained load level (given as an average bond shear stress in MPa), and (4) repetition number (in cases where two or more identical tests were performed). For example, a specimen with a CFRP tendon tested under SL conditions at a sustained average bond stress of 2.6 MPa (0.38 ksi) would be denoted as C-SL-2.6. The first column of Table 1 provides a summary of all of the tests that were performed for the current study.

PULLOUT TEST RESULTS AND DISCUSSIONS

AT (Ambient Temperature)

The CFRP pullout specimen tested under AT conditions failed by slipping at the bond interface between the sand coating and the tendon, at a bond stress of 5.3 MPa (0.77 ksi). After failure, pullout continued at a constant rate, and a remnant bond strength capacity of 4.09 MPa (0.59 ksi) was measured (Figure 5).

The steel pullout specimen tested under AT conditions failed by tensile rupture (fracture) of the steel reinforcement at the loaded end outside the cylinder, as shown in Figure 6 (i.e. the AT bond capacity for steel was greater than the tensile strength of the steel wire). The 16.6 MPa (2.41 ksi) maximum bond stress achieved thus represents the steel bar’s tensile strength rather than a true bond strength.

IL (Increased Loading)

CFRP pullout specimens tested under IL conditions failed (recall, after the load was increased above the sustained load that was applied during heating) by slippage at the bond interface between the sand coating and the tendon. In all IL tests on CFRP tendons the temperature at the concrete-to-CFRP interface was greater than 164 °C (327 °F) during the final load ramp phase. Three CFRP pullout specimens were tested under IL conditions, with initial sustained bond stresses of 0.7 MPa (0.10 ksi), 1.5 MPa (0.22 ksi) and 1.8 MPa (0.26 ksi). The bond strength at elevated temperature, measured after further increase of the load, was between 73% and 96% of the bond strength at ambient temperature, as shown in Figure 5. No correlation between the initial sustained bond stress and the bond strength at elevated temperature was apparent. After failure occurred, pullout continued at a constant rate with a smooth and gradual reduction in bond stress and a remnant bond strength tending to values above 2.2 MPa (0.32 ksi) (see Figure 5).

Steel pullout specimens tested under IL conditions failed (after load was increased) by slip at the bond interface between the steel wire and the concrete. In all IL tests on steel wires the temperature of the concrete-to-steel interface was greater than 160 °C (320 °F) at failure. Four steel pullout specimens were tested under IL conditions, with initial sustained bond stresses of 6.1 MPa (0.88 ksi) and 7.6 MPa (1.10 ksi); two specimens were tested at each initial bond stress. The bond strength at elevated temperature was between 71% and 86% of the maximum bond stress measured at ambient temperature (recall that this was restricted by the tensile strength of the steel reinforcement), as shown in Figure 6. No correlation between the initial sustained bond stress and the bond strength at elevated temperature was observed. After failure occurred, pullout continued at a constant rate with sudden repeated drops in bond stress (see Figure 6). The differences in the post-peak response of the steel specimens versus the CFRP specimens clearly illustrate the significant differences in the bond mechanisms (strength and stiffness degradation) for the two reinforcements, particularly at elevated temperature.

SL (Sustained Loading)

CFRP pullout specimens tested under SL conditions failed during heating while under sustained load. Failure occurred by slip at the bond interface between the sand coating and the tendon. A correlation between the sustained load level and the temperature at which failure occurred was observed; as the sustained load was increased, failure occurred at successively lower temperatures (see Figure 8(a)). The sustained load was maintained for a short time during the bond slipping process, although a drop of the bond strength (corresponding to total failure of the bond) occurred for high slips; this is shown by comparing Figure 7(b) and 7(c). Because the crosshead displacement was configured in a load control mode, sudden large slips occurred at failure. An apparent remnant bond strength capacity was measured, which was affected by the conditions in which failure occurred (see Figure 9). These values of remnant bond strength are considerably lower than those measured for the IL tests in which failure occurred under displacement control mode (refer to Figure 5).

The CFRP pullout specimen tested under SL conditions with a sustained load of 3.3 MPa (0.48 ksi) (C-SL-3.3), failed by slip at the bond interface between the sand coat and the tendon within seconds of the heating blanket being turned on (Table 1). This was attributed to localized bond stress increases due to differential longitudinal thermal expansion between the concrete and the CFRP tendon, and possibly also to short term creep deformation due to the relatively high sustained load (62% of the bond strength at ambient temperature).

For the other three CFRP pullout specimens under SL conditions, with sustained bond stresses of 2.2, 2.6 and 3.0 MPa (0.32, 0.38 and 0.44 ksi), the 'failure' temperature was defined using digital image correlation analysis. During these tests, the load was maintained at the predefined level (bond stress) in a load control mode (see Figure 7(c)). Small negative loaded end slips were measured during heating of the specimens (see Figure 7(b)). These slips were considered to be produced by three factors which occurred as a consequence of the testing method and which do not represent true slip of the reinforcement within the cylinders [22]. These are: (1) longitudinal thermal expansion of the reinforcing bars, (2) longitudinal thermal expansion of the concrete cylinder, and (3) crosshead displacement produced by the testing frame operating load control mode and attempting to maintain the initial predefined bond stress. Figure 7(b) shows this behavior for a typical CFRP SL test, where negative slips are observed after the heating blanket is turned on, in this case beginning at minute 15 (see Figure 7(a)). Because of the complexity of this phenomenon, the bond failure temperature of the specimen was arbitrarily defined by a positive slip increment of 0.05 mm (1.97e^{-3} in.) from the temperature at which the slip was at its lowest value after the instant that the heating began. This definition is shown schematically in Figure 8(a). Figure 8(b) includes a curve showing the reduction in storage modulus (effectively the flexural elastic modulus) of the CFRP's epoxy resin matrix with temperature, as determined from dynamic mechanical analysis (discussed later).

It is noteworthy that CFRP pullout tests under SL conditions failed at bar temperatures in the range of 98-111°C (208-232 °F), with sustained bond stresses above 2.2 MPa (0.32 ksi), whereas tests under IL conditions, which had sustained bond stresses below 1.8 MPa (0.26 ksi) did not fail as temperature increased but only after further loading occurred at elevated temperature. Counter-intuitively, a bond strength of 3.9-5.1 MPa (0.57-0.74 ksi) was achieved for the IL conditions, despite the fact that the bar at temperatures were above 164 °C (327 °F) in all cases. It is hypothesized that the CFRP bars' high transverse coefficient of thermal expansion, which is about three times that of concrete [24], increased the bond strength capacity of the CFRP bars at elevated temperature by a confining effect from restrained lateral thermal expansion of the bar. Conversely, previous researchers [25], [26] have shown that transverse thermal expansion of an FRP bar in concrete can generate a drop of the bond strength capacity due to bursting of the surrounding concrete. Additional research on the impacts of transverse differential thermal expansion of FRP bars in concrete it therefore warranted.

Steel pullout specimens tested under SL conditions failed during heating when the bond stress was sustained at 9.1 MPa (1.32 ksi). For these tests, failure occurred by transverse splitting of the concrete cylinders, only six minutes after the heating wrap was turned on, when the thermal gradient between the hot concrete surface and the still cold reinforcement was at a high level (Figure 11). It is hypothesized that the thermal gradient within the concrete cylinder generated differential thermal stresses which, when superimposed on the bursting stresses in the concrete produced by the pullout effect of the bar, precipitated the splitting failure. Since failure of these specimens occurred while under a load control mode, a rapid increase of slip occurred immediately after failure (see Figure 10). After reaching the initial bond stress peak failure progressed by bond slip followed by successively smaller bond stress peaks as the steel bars pulled out of the cylinders. The distance between successive bond stress peaks was approximately the same as the distance between deformation lugs on the steel prestressing wire, which caused the observed stick-slip response. It is likely that bond failure would have been total on first cracking if the concrete had not incorporated polypropylene fibers which bridged the splitting cracks and prevented global failure of the cylinders.

For all of the pullout tests under SL conditions, neither the digital image correlation technique nor the LPs were able to measure the large slips which occurred immediately after failure. Bond stress versus crosshead displacement data is therefore shown in Figure 9 and Figure 10 to illustrate the failure behavior of these specimens.

Realistic confinement of the bonded lengths which would be encountered in large scale structural elements was not accurately reproduced in the pullout tests performed in the current study, so that the practical importance of the splitting phenomenon observed for the steel pullout specimens for real structures in fire remains unknown. Another potentially important factor is that, for prestressed concrete, the reinforcement is subjected to the Hoyer effect, which results from the prestressing strand swelling as a result of prestress loss at transfer of prestressing force to the concrete and Poisson's effects [27].

The vapor pressure issues experienced by Katz et al. [6] (discussed previously) were not observed in the current experimental investigation. This may be because polypropylene micro-fiber reinforced concrete was used, which, as mentioned by Khoury [28], allowed vapor pressures to dissipate without affecting bond performance. Additional research on the effects of polypropylene fibers in concrete at elevated temperature is needed.

TRANSIENT ELEVATED TEMPERATURE TENSILE STRENGTH TESTS

As already noted, for prestressed steel reinforcement it is widely assumed that the tensile strength of the reinforcement is more critical in fire than any reductions in bond strength that might be experienced. To partially verify this assumption, and also to demonstrate that the opposite is true when using bonded FRP prestressing reinforcements (i.e. bond strength reductions at elevated temperature are more critical than tensile strength reductions for FRP bars), transient high temperature tensile strength tests were performed.

The tensile strength of both steel and CFRP reinforcements is expected to be reduced at elevated temperatures. For cold-drawn steel prestressing wire, the relationship between tensile strength and temperature is relatively well established and is available, for example, in Eurocode 2 [7]. However, the effects of elevated temperature on the tensile strength of the CFRP prestressing tendons used in the current study are not known. Transient elevated temperature tensile strength tests (load-then-heat) were performed for both the steel wires and the CFRP tendons at sustained stress levels of 800 MPa (116 ksi), 1000 MPa (145 ksi), and 1200 MPa (174 ksi); these represent a realistic range of in-service stress for pretensioning applications with these materials. The wires/tendons were loaded under displacement control at ambient temperature to the predefined tensile stress level [1], and the load was maintained while the tendon was heated at 10 °C/min (18 °F/min) until tensile failure occurred. Note that the anchorages were protected from exposure to elevated temperature throughout the testing (the grips were outside the thermal chamber).

Figure 12(a) shows the elevated temperature tensile strength results for the steel prestressing wires. Also included in this figure is the ultimate tensile strength reduction curve recommended by Eurocode 2 [7] for Class A cold-drawn prestressing steel. It is clear from this figure that prestressing steel suffers considerable reductions in tensile strength at elevated temperature. This figure also shows, as expected, that the Eurocode 2 [7] tensile strength reduction recommendations are conservative (by about 50 °C (122 °F) to 100 °C (212 °F)) with respect to the testing methods and materials used in the current study.

Figure 12(b) shows the elevated temperature tensile strength results for the CFRP prestressing tendons. Also included in this figure are the results from a thermogravimetric analysis (TGA) performed on the CFRP material; this is explained and discussed in the following section. Figure 12(b) shows that the CFRP tendon also experiences considerable reductions in tensile strength with exposure to elevated temperature. On average, the CFRP tendon is slightly more sensitive to elevated temperature than the steel wire, experiencing failure at temperatures about 10 °C (50 °F) to 50 °C (122 °F) lower than the steel at the same stress level. Since the tensile strength of the CFRP is clearly less sensitive to the effects of elevated temperature than its bond strength, the conventional design methodologies applied to steel prestressed concrete cannot be applied to CFRP prestressed concrete (indeed, there is some evidence herein that the conventional approach for steel prestressing may require re-evaluation, since the bond of steel prestressing wire to concrete is not insensitive to elevated temperature exposure).

MICROMECHANICAL ANALYSIS OF CFRP TENDONS

Dynamic mechanical analysis (DMA)

It is widely considered that upper temperature limits for FRP materials used in infrastructure applications should be defined in terms of the glass transition temperature (T_g) of the polymer resin matrix used in the FRPs' fabrication [29]. While there are many different test methods and techniques by which the T_g for a given polymer can be determined, for the polymers used in infrastructure applications T_g is most commonly defined using dynamic mechanical analysis (DMA), with T_g defined using a T_g-onset definition. This definition is shown in Figure 13, where the normalized retained storage modulus ($E(T)/E_{AT}$) of the specimen (essentially the normalized retained flexural elastic modulus of the epoxy resin) is plotted versus temperature for the CFRP bars used in the current study. The T_g-onset value for the CFRP tendons was 121 °C (250 °F). It should be noted that a variety of other definitions for T_g may also be used with a DMA analysis (Tan δ peak, for instance) depending on the particular industry and jurisdiction.

The DMA analysis shown in Figure 12 was executed using a dynamic mechanical analyzer [30] which measures the viscoelastic response of materials as a function of temperature. A rectangular cross section of 2x4 mm (0.079x0.147 in.), 15 mm (0.59 in.) long CFRP specimen was cut from a CFRP tendon and subjected to single cantilever sinusoidal loading. The loading frequency was 1 Hz and the heating rate was 3 °C/min (5 °F/min). The storage modulus, loss modulus, and Tan δ values were determined as functions of temperature. The storage modulus represents the elastic response of the specimen, whereas the loss modulus represents the viscous response. δ represents the phase shift angle between the elastic and viscous responses, and is also sometimes used to define T_g for a given polymer by taking the peak Tan δ value [31]. From the DMA analysis, the T_g -Tan δ for the CFRP bars used in the current study was determined to be 148 °C (298 °F).

It was desired to determine if there is any obvious relationship between either T_g or the measured reductions in the storage modulus of the resin and the bond strength (or tensile strength) of the CFRP prestressing tendons. However, because other factors, such as the CFRP's and concrete's transverse and longitudinal thermal expansion, impacted the results it is difficult to draw general conclusions in this regard. Nonetheless, Table 1 provides the $E(T)/E_{AT}$ values for the CFRP at the instant that bond failure was initiated (using the bond failure definition shown in Figure 8(a)). These data shows that the relationship between polymer resin stiffness and bond strength is more complex than can be defined simply with reference to T_g. For instance, CFRP pullout specimens tested under IL conditions failed at bond stresses between 3.9 MPa (0.57 ksi) and 5.1 MPa (0.74 ksi) when the normalized retained storage modulus was between 0.35 and 0.36, whereas CFRP pullout specimens tested un SL conditions failed at bond stresses between 2.2 MPa (0.32 ksi) and 3.3 MPa (0.48 ksi) when the normalized retained storage modulus was between 0.90 and 1.00. It is clear that additional research is needed to better understand this behavior.

Thermogravimetric analysis (TGA)

Previous research has shown that while the polymer resins used in FRP manufacturing are sensitive to temperatures in the range of 45-100 °C (113-212 °F) [3], carbon fibers are essentially insensitive to temperatures as high as 600 °C (1112 °F). However, interaction between the fibers in an FRP component is essential to provide stress transfer between individual fibers and to prevent failure of the bulk material upon loading. Thermogravimetric analysis (mass loss with temperature) was performed on a small, unstressed 46.3 mg (1.63 oz) specimen cut from one of the CFRP tendons. The goal of the testing was to determine if the temperature of resin decomposition, T_d-onset in Figure 13, could be used to provide an indication of the temperatures which may lead to tensile failure of the CFRP tendons under service prestress levels between 800 and 1200 MPa (116 and 174 ksi).

Figure 13 shows the TGA test, which was performed in an inert N_2 environment up to a temperature of 600 °C (1112 °F) and in air afterwards. A heating rate of 20 °C/min (36 °F/min) was used. The thermal degradation is correlated and displayed as the normalized reduction of the specimen's mass [32]. It can be observed that the thermal degradation of the epoxy resin starts at approximately 290 °C (554 °F) and ends at approximately 600 °C (1112 °F), while the oxidation of the carbon fibers begins at about 620 °C (1148 °F) and continues until the test finishes at 900 °C (1652 °F). The TGA data are also plotted together with the CFRP tension test data in Figure 12(b). Comparison of the measured T_d-onset value of 329 °C (624 °F) against the temperature to cause tensile rupture suggests that T_d-onset provides a reasonable indication of the critical temperature for the tendons.

Clearly, this observation only holds for the stress levels and testing procedures used in the current study, and additional research on a variety of FRP materials and sustained stress levels is needed to verify the correlation.

CONCLUSIONS AND RECOMMENDATIONS

Many aspects of bond performance at elevated temperature, for both FRP and for steel reinforcing bars (prestressed and non-prestressed), remain poorly understood and require additional investigation. The thermal and mechanical conditions to which a pullout specimen is subjected at elevated temperatures were found to be far more complex than has previously been reported in the literature. A wide variety of interrelated factors play roles in the bond strength response at elevated temperature, including: steam pressure, lateral and longitudinal thermal gradients in the concrete, lateral and longitudinal differential thermal expansion between the concrete and the reinforcement, prestress levels and resulting Poisson's effects in the reinforcement, transverse and splitting cracking in the concrete, and the type and surface condition (i.e. bonding mechanisms) of the reinforcement.

For the CFRP prestressing tendons, reductions in bond strength appear to be more critical than loss of tensile strength of the reinforcement at elevated temperature, such that the fire resistance of CFRP prestressed concrete elements is likely to be governed by provision of a cool anchorage for the CFRP tendons. High bond stress values led to lower bond failure temperatures (below the resin T_g by any definition in all cases). That being said, the results show that it is difficult to provide detailed guidance, with pullout experimentation, on the limiting temperatures which should be imposed to ensure adequate bond strength during heating.

The bond strength of the steel prestressing wires appears to be less sensitive to elevated temperatures than the FRP reinforcement, although bond strength reductions in the order of 17-29% were observed at temperatures of only 160 ºC (250 ºF). The pullout specimens with steel prestressing wire, which in general had higher sustained bond stress values than the FRP tendons during heating, were particularly susceptible to splitting cracking in the early stages of heating. The development of these splitting cracks is highly relevant to the fire-safe design of steel-prestressed concrete structures and should receive additional research attention.

The tensile strength of the CFRP prestressing tendons was slightly more sensitive to elevated temperature than that of the steel prestressing wire, experiencing failure at temperatures about 10 °C (50 °F) to 50 °C (122 °F) lower than the steel wire at the same sustained stress level. This shows that the strength of CFRP is not vastly more sensitive to elevated temperature than is steel. The onset resin degradation temperature, T_d-onset, appears to be a reasonable proxy for the critical temperature of the CFRP tendons tested in the current paper (on the basis of tensile strength only).

Considerable additional testing is needed to fully understand the complexities of the interactions between stress, time, and temperature that eventually lead to bond failure in a pullout test executed at elevated temperatures, both for steel and CFRP prestressing reinforcement.

Bond of prestressed reinforcement is considered in some structural fire design codes (e.g. [7]), although rules are not explicitly given. Additional research is required both for FRP and steel bars so that practical guidance in this area can be given to designers.

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Table 1 — Selected details of pullout test program and results

Test notations*	Prestressing Condition				Failure Condition									Remnant bond strength	
	Bond Stress		Tensile Stress		Bond Stress		Tensile Stress		Temperature				Resin Matrix Storage Modulus		
									Bar		Blanket				
	MPa	(ksi)	MPa	(ksi)	MPa	(ksi)	MPa	(ksi)	°C	(°F)	°C	(°F)		MPa	(ksi)
C – AT	-		-		5.3	(0.77)	633	(91.81)	21	(70)	24	(75)	1.00	4.09	(0.59)
C – IL – 0.7	0.7	(0.10)	87	(12.62)	3.9	(0.57)	461	(66.86)	166	(331)	182	(360)	0.35	2.26	(0.33)
C – IL – 1.5	1.5	(0.22)	175	(25.38)	5.1	(0.74)	601	(87.17)	166	(331)	182	(360)	0.35	2.21	(0.32)
C – IL – 1.8	1.8	(0.26)	218	(31.62)	4.3	(0.62)	513	(74.40)	164	(327)	185	(365)	0.36	2.29	(0.33)
C – SL – 2.2	2.2	(0.32)	262	(38.00)	-		-		111	(232)	169	(336)	0.90	0.65	(0.09)
C – SL – 2.6	2.6	(0.38)	306	(44.38)	-		-		108	(226)	155	(311)	0.91	0.89	(0.13)
C – SL – 3.0	3.0	(0.44)	349	(50.62)	-		-		98	(208)	148	(298)	0.94	0.92	(0.13)
C – SL – 3.3	3.3	(0.48)	393	(57.00)	-		-		21	(70)	76	(169)	1.00	-	
S – AT	-		-		16.6	(2.41)	1774	(257.30)	22	(72)	24	(75)	-	-	
S – IL – 6.1 (1)	6.1	(0.88)	648	(93.98)	11.8	(1.71)	1257	(182.31)	164	(327)	184	(363)	-	-	
S – IL – 6.1 (2)	6.1	(0.88)	648	(93.98)	13.5	(1.96)	1434	(207.98)	160	(320)	178	(352)	-	-	
S – IL – 7.6 (1)	7.6	(1.10)	810	(117.48)	12.8	(1.86)	1365	(197.98)	170	(338)	192	(378)	-	-	
S – IL – 7.6 (2)	7.6	(1.10)	810	(117.48)	13.7	(1.99)	1460	(211.76)	162	(324)	179	(354)	-	-	
S – IL – 9.1 (1)	9.1	(1.32)	972	(140.98)	-		-		31	(88)	155	(311)	-	-	
S – IL – 9.1 (2)	9.1	(1.32)	972	(140.98)	-		-		25	(77)	118	(244)	-	-	
S – IL – 9.1 (3)	9.1	(1.32)	972	(140.98)	-		-		25	(77)	105	(221)	-	-	

* C: CFRP tendon, S: steel wire, AT: ambient temperature, IL: incremented load, SL: sustained load.

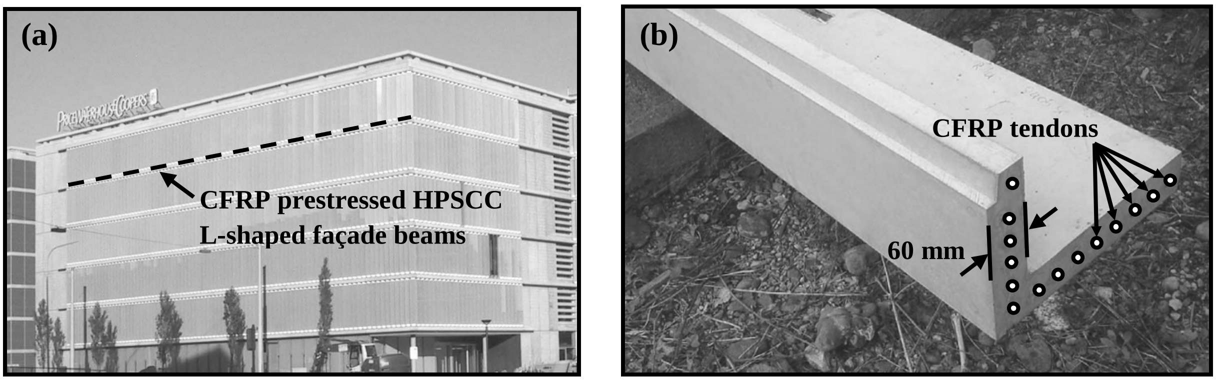

Figure 1 — (a) Application of CFRP prestressed HPSCC L-shaped facade beams and (b) corresponding cross section.

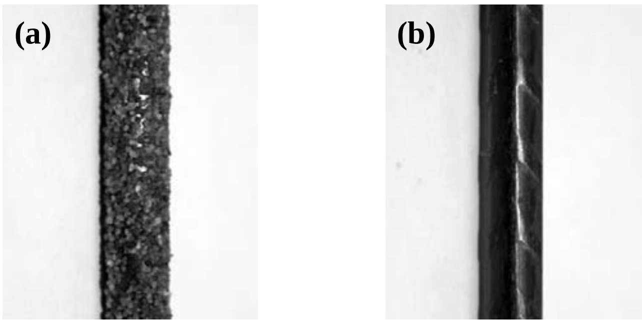

Figure 2 — (a) CFRP prestressing tendons and (b) deformed steel prestressing wire.

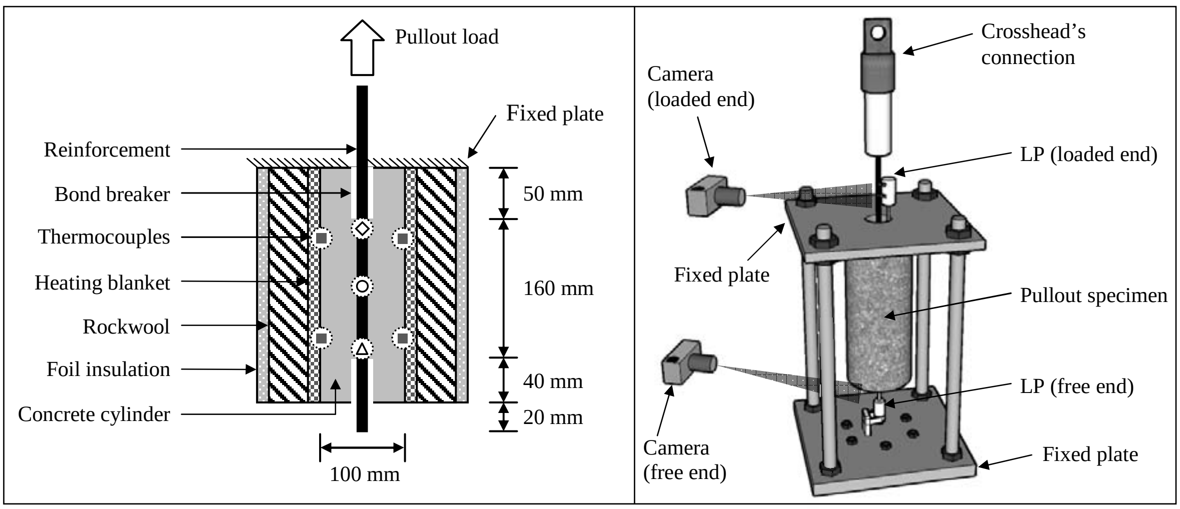

Figure 3 — Schematic of pullout test specimens and experimental setup for pullout tests.

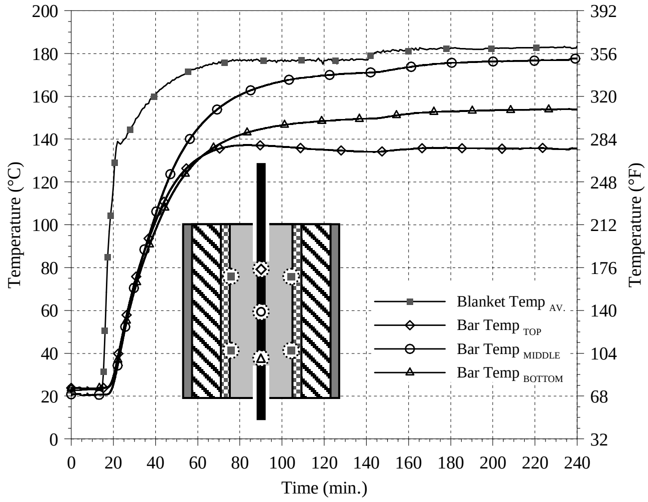

Figure 4 — Typical recorded temperatures in a pullout specimen during heating.

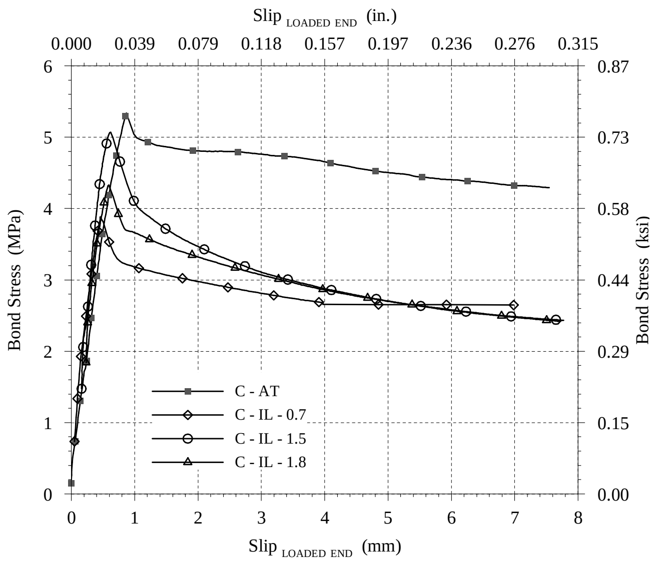

Figure 5 — Bond stress versus loaded end slip for AT and IL type tests with CFRP reinforcement.

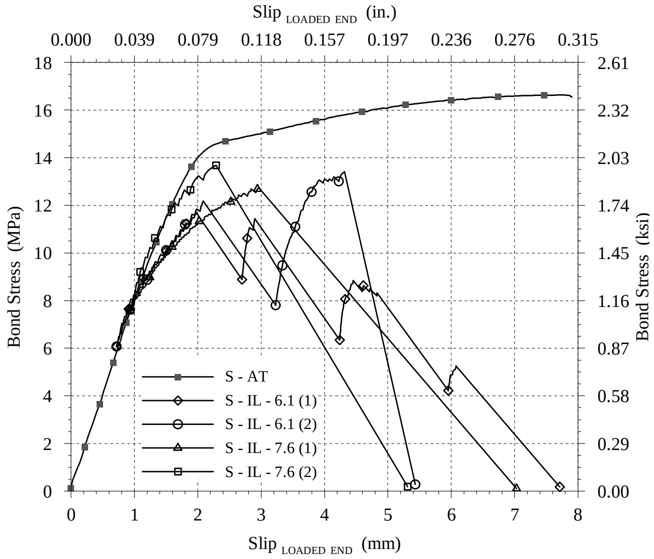

Figure 6 — Bond stress versus loaded end slip for AT and IL type tests with steel reinforcement.

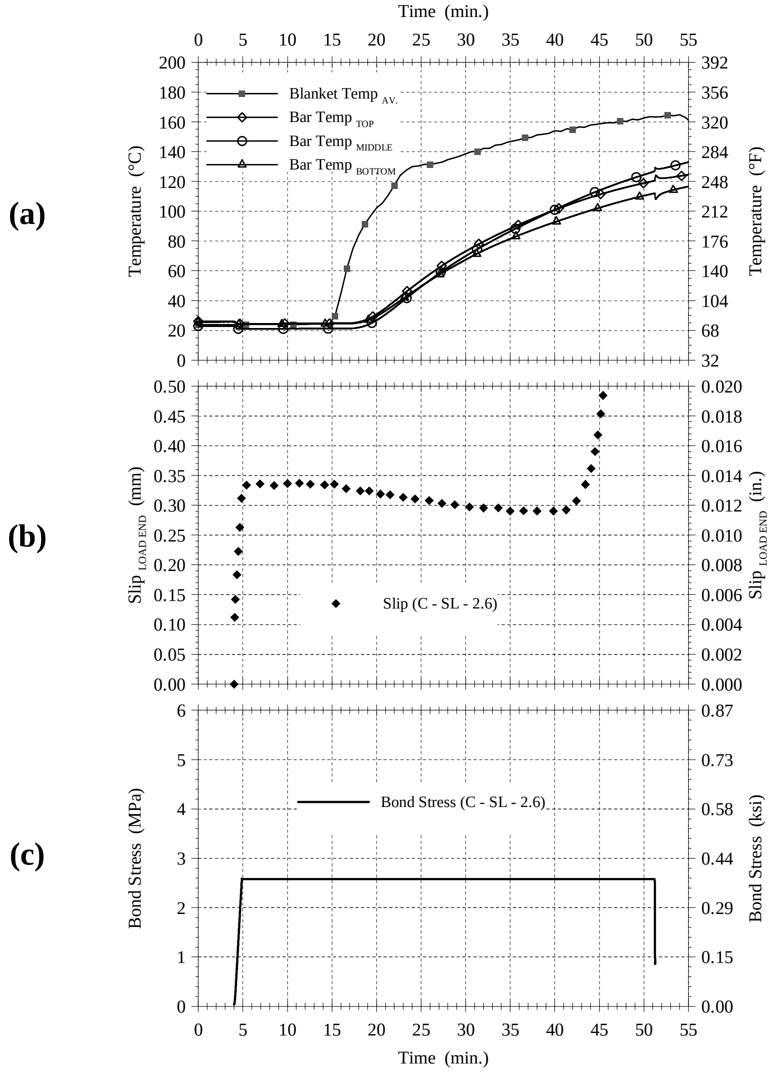

Figure 7 — Behavior of a typical CFRP specimen tested under SL conditions: (a) recorded temperatures (b) loaded end slip and (c) bond stress (all versus time in minutes).

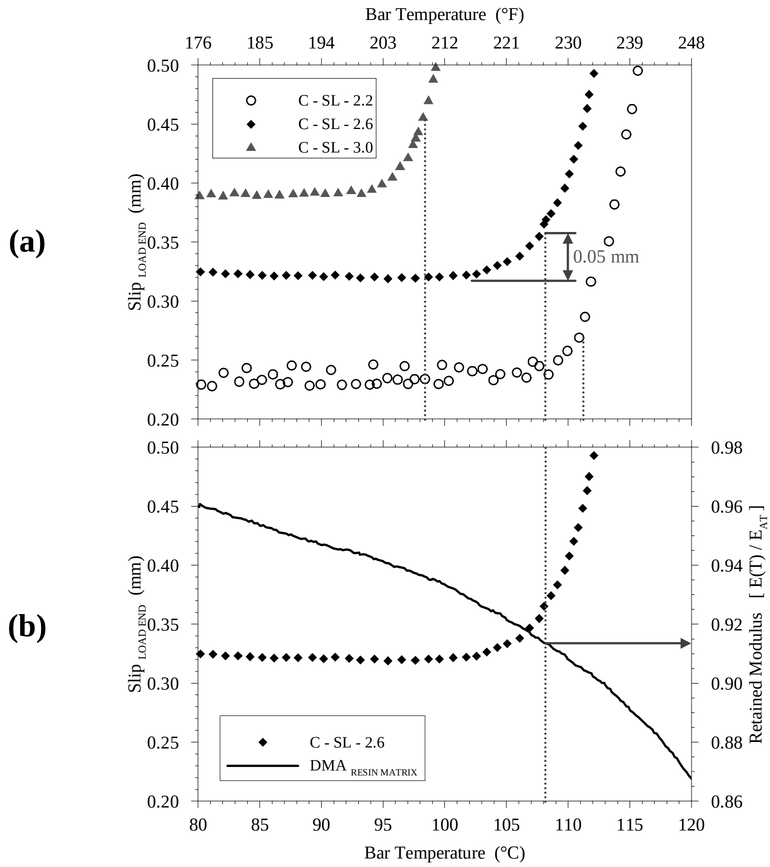

Figure 8 — (a) Close-up view of loaded end slip versus bar temperature for SL tests on CFRP tendons and (b) schematic showing how bond failure temperature was defined for a typical SL test with CFRP; also including storage modulus data from dynamic mechanical analysis (DMA).

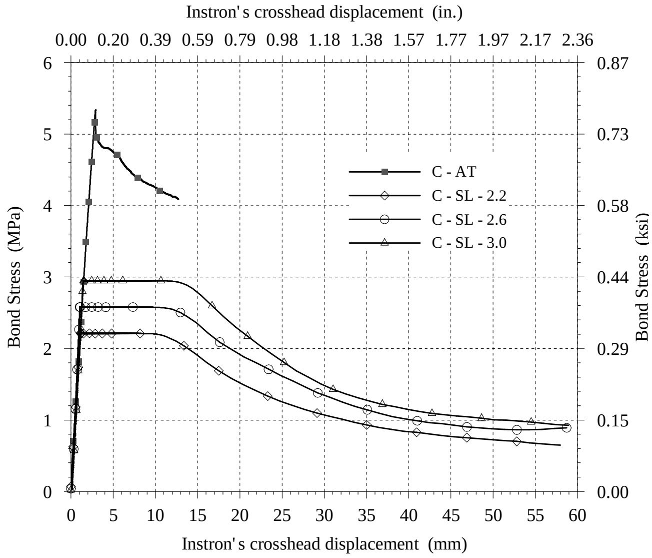

Figure 9 — Bond stress versus crosshead displacement behavior for SL tests with CFRP reinforcement.

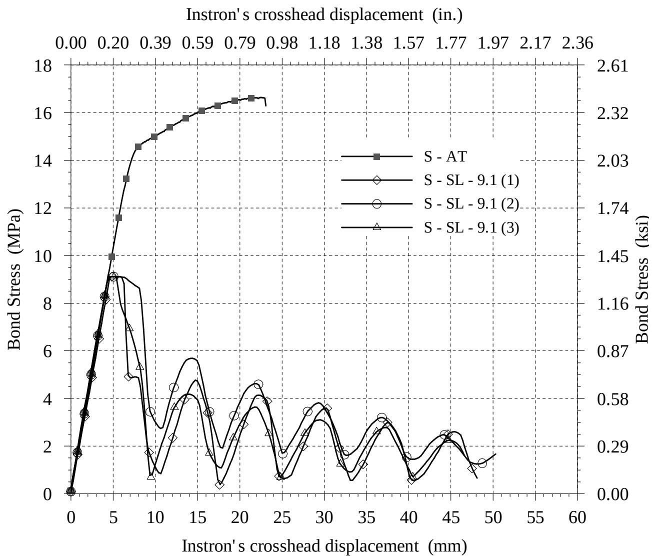

Figure 10 — Bond stress versus crosshead displacement behavior for SL tests with steel reinforcement.

Figure 11 — Typical transverse splitting of the concrete cylinders for samples tested under S-SL-9.1 conditions.

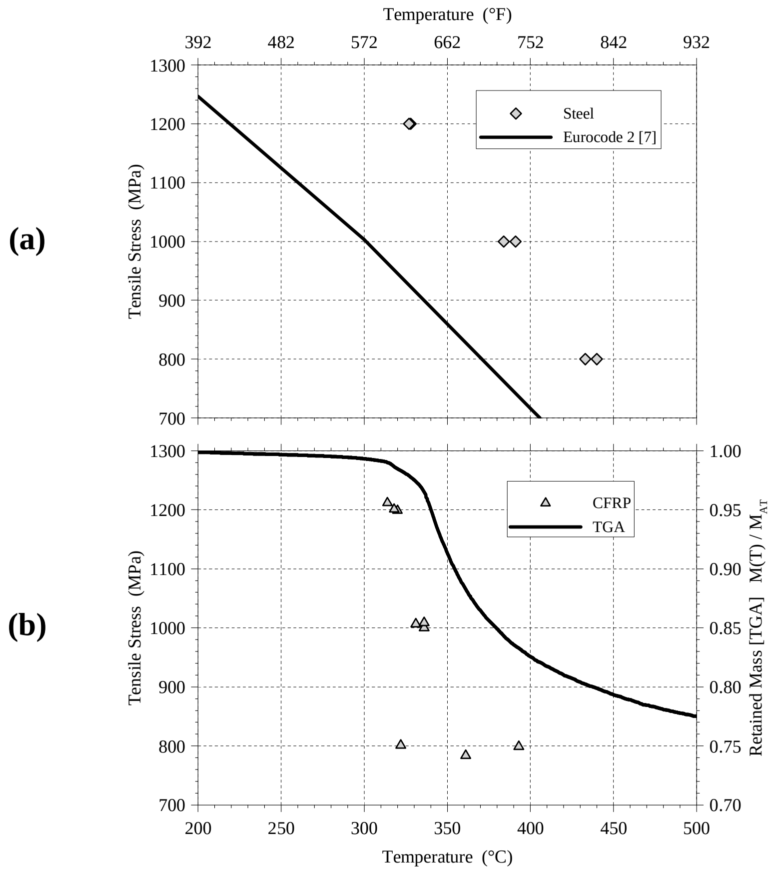

Figure 12 — Transient elevated temperature tensile strength test results for (a) steel and (b) CFRP reinforcing bars, compared with Eurocode [7] and TGA results, respectively.

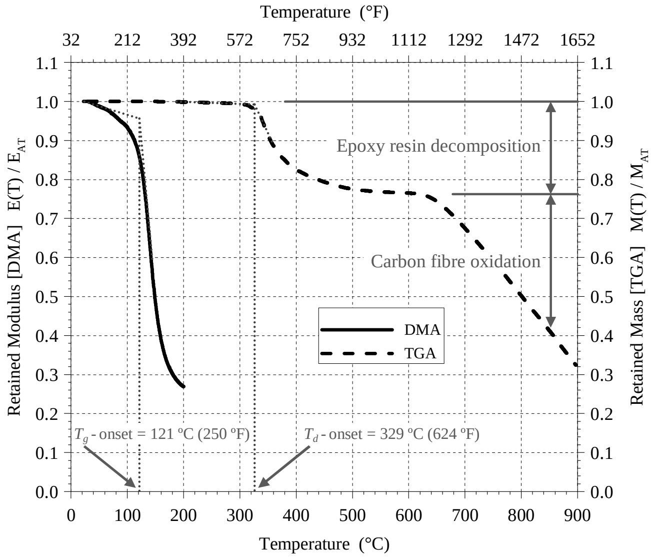

Figure 13 — DMA and TGA test results.